INDUSTRIAL AND CIVIL ENGINEERING AND ECONOMICS

DOI 10.15826/rjcst.2018.1.001 УДК 627.8

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EVALUATING DEGREE OF COMPACTION OF LEVEES USING CONE PENETRATION TESTING

Abstract. Permeability and strength parameters of compacted soils (i. e. levees as well as other earthworks) may be correlated to the degree of compaction. Since the use of conventional and recent testing methods for the assessment of density and water content of earthworks, under construction, cannot be applied to existing levees, an expeditious and accurate method for the assessment of the degree of compaction of existing and new levees, after their completion, appears extremely useful. The purpose of this research is to develop a simple tool for the assessment of the degree of compaction of "compacted", partially saturated, fine-grained soils. This paper illustrates the proposed method which combines in situ testing like electric CPT or CPTu with laboratory penetration testing performed with a mini-cone in a calibration chamber (CC).

Keywords: permeability, strength parameters, compacted soils, levees, earthworks, under construction, in situ testing, laboratory penetration testing, calibration chamber.

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ОЦЕНКА СТЕПЕНИ УПЛОТНЕНИЯ ДАМБ МЕТОДОМ УДАРНОГО КОНУСА

Аннотация. Водопроницаемость и показатели прочности уплотненных грунтов (дамб и других земляных сооружений) могут быть соотнесены со степенью уплотнения. Традиционные и используемые в настоящее время методы определения плотности и содержания воды в грунтах земляных сооружений, находящихся в процессе строительства, не могут быть применены для существующих дамб. Вследствие этого быстрый и точный метод для оценки степени уплотнения существующих и новых дамб после завершения их строительства является чрезвычайно полезным. Цель этого исследования — разработка простого метода для оценки степени уплотнения «уплотненных», частично водонасыщенных, мелкозернистых почв. В статье описывается метод, сочетающий испытания в условиях строительной площадки (зондирование) и лабораторные испытания с использованием мини-конуса в калибровочной камере.

Ключевые слова: проницаемость, прочностные параметры, уплотненные грунты, дамбы, грунты в процессе строительства, испытания в условиях строительной площадки, лабораторные испытания методом ударного конуса, калибровочная камера.

Introduction

The assessment of the safety factor against possible Ultimate Limit States (ULS) of existing levees requires, at least, the knowledge of strength and permeability parameters. On the other hand, it is well recognized that these parameters mainly depend on the degree of compaction and the degree of saturation (see as an example [72]). Therefore, the assessment of soil density and water content can contribute to a correct estimate of the required parameters.

Unfortunately, most of the existing levees were constructed several centuries ago by using poor techniques and poor materials (i. e. on-site available soils). Moreover, the construction details of such existing levees are unknown.

The use of both conventional and recent testing methods for the assessment of density and water content of earthworks, under construction, cannot be applied to existing levees. Indeed, the Rubber Balloon Method (ASTM D2167) [6], the Sand Cone Method (ASTM

D1556) [7], the Time Domain Reflectometry (ASTM D6780) [8] and the Nuclear Methods (ASTM D6938) [9] are only applicable at shallow depths. On the other hand, the use of specially equipped piezocones for electrical resistivity measurements [40, 20] is not very popular and its application is restricted to fully saturated soils. Also nuclear density probes [57, 73] are not very popular and their use is mainly restricted to offshore applications.

As far as the construction of new embankments is concerned, the common practice (at least in Italy) essentially requires the following design — prescriptions and controls during the construction stage:

- soil type (generally referring to AASTHO M145, 1991 [1]);
- compaction method (equipment, number of passes, layer height);
- required dry density and water content. These are usually inferred from Standard Proctor (ASTM D698 [10]) or Modified Proctor (ASTM D1557 [11]) Methods. The required dry unit weight is usually defined as percentage of the optimum dry density;
- typical controls, during the construction stage, are based on in situ density tests or plate load tests (PLTs) which are very time consuming. For these controls, the above mentioned “shallow depth” methods are also applicable.

In practice, design criteria conform to those adopted for road embankments. The poor attention devoted in the past to the design and construction of levees depends on various reasons. Usually, levees are in unsaturated conditions even during floods because of the short duration of these events. On the other hand, stability analyses of levees are generally carried out under the condition of steady state flow in a saturated medium. Therefore, usual stability analyses neglect the beneficial effect of suction (see as an example [25, 27]). Moreover, most of the existing levees have been constructed before Second World War. Since at their construction time, huge floodplain areas were available, therefore only the main levees, devoted to the hydraulic protection of urbanised areas, were designed to resist to floods, whereas levees of minor importance, constructed for the protection of the country areas, were often made deliberately destructible during a flood events.

The considerable and rapid urbanization that occurred, especially in western countries, after Second World War, made the safety assessment of such minor levees necessary. Since budgets for levees refurbishments are limited, a priority list becomes mandatory. At the same time, adverse weather conditions are becoming more and more frequent because of global climatic changes. Particularly adverse climatic conditions (repeated floods within 10–15 days, very prolonged rain periods, very intense rainfalls, etc.) can lead to an almost complete saturation of the levees and cause their failures [27, 28, 69]. As a matter of fact, between 1998 and 2009, European Union suffered over 213 major damaging floods, including the catastrophic floods along the Danube and Elbe rivers in summer 2002. Severe floods in 2005 further reinforced the need for concerted action. Between 1998 and 2009, floods in Europe have caused some 1126 deaths, the displacement of about half a million people and at least €52 billion in insured economic losses (http://www.eea.europa.eu/themes/water/water-resources/floods).

As already stated, both permeability and strength of new and existing levees are affected by the degree of compaction as well as by the saturation degree. Therefore an expeditious and accurate method for the assessment of the degree of compaction of existing and new levees appears extremely useful.

The proposed method combines in situ testing like electric CPT or CPTu with laboratory testing, i. e. penetration testing with a mini-cone in a Calibration Chamber (CC).

Calibration chambers review and hypotheses

Calibration Chambers (CCs) with large diameter ($D_{CC} \geqslant 1.2$ m) have been used in pioneering works with standard CPT testing in sand samples [16, 23, 67, 74]. This choice was dictated by the fact that the D_{CC}/d_C ratio, with d_C cone diameter, should be large to consider the soil model as an infinite medium [59]. The appropriate value of the D_{CC}/d_C ratio is not a constant but mainly depends on sand type, relative density and boundary conditions. In any case, the use of small CCs has become more and more popular especially after the contribution of [37] showing the capability of the dynamic control of horizontal pressure (see also the papers by [41, 42]). Several researchers have developed small CCs with mini-cone [2, 30, 34, 46, 47, 48. 62].

While all the above mentioned studies employed mini-cones in 1g conditions, other mini-cones were also been developed for centrifuge testing (se as an early example [18]).

The above mentioned researches have been carried out for different purposes and very advanced mini-cones were realized. The purpose of the present research is that of developing a simple tool for the assessment of the degree of compaction of “compacted”, partially saturated, fine-grained soils. A complete and exhaustive review of previous of CPT testing in CCs is out of the scope of the present research.

A mini calibration chamber with a diameter of 320 mm and a mini cone with a diameter of 8 mm was developed in order to perform penetration tests on Ticino sand and four different types of fine-grained soils. The scope of the tests was to assess the influential factors controlling the tip resistance and to define empirical correlations between tip resistance and soil dry density or degree of compaction. Tests on the well known Ticino sand were carried out only for a preliminary check of the equipment.

A similar procedure is described in the technical standards by [3, 4]. This procedure is applied to coarse grained soils and requires the construction of a trial embankment (physical soil model) and the performance of dynamic penetration tests. As a results a reference "penetrogramme" (i. e. displacement per blow vs. depth) is obtained from the experiments. The standards also state the criteria for the acceptance of the in situ controls in comparison to the design "penetrogramme". According to [65, 66], this methodology should be applied to the control of the compaction degree of trenches.

The proposed method is based on the following considerations and assumptions.

On the whole, the tip resistance values may be a function of many factors, including: the soil type; the degree of compaction; the degree of saturation when compacted; the degree of saturation during penetration; the penetration rate; the time elapsed after the levee construction.

Four assumptions are made.

First assumption: the tip resistances of a standard cone (d = 35.7 mm) and a mini-cone (d_C = 8 mm) are the same irrespective of the cone diameter when carried out in the same soil under the same conditions. This hypothesis involves two different aspects. The first is the ratio between the cone diameter and the grain size of the soil. This aspect is discussed with the fourth hypothesis. The second aspect is related to the normalized penetration rate that, according to [24, 76], is expressed as:

$$V = \frac{v \cdot d}{c_v} \qquad (1)$$

where V — normalized penetration rate; d — cone diameter, v — penetration rate, c_v — coefficient of consolidation.

It is evident that for the mini-cone penetration occurs at a lower normalized penetration rate. More specifically, the mini-cone has a normalized velocity four times smaller than that of a standard cone. According to many researchers, higher tip resistances should be measured at lower normalized penetration rates, especially in the case of saturated silty clay (see as an example [17, 51, 63]).

In any case, the correctness of the hypothesis, for the soils under consideration, has been experimentally verified by performing at close distances 4 standard and 4 mini-cone tests in the Calendasco site (Piacenza, Italy). The tested soil is an unsaturated silt mixture. Fig. 1 shows the upper and lower envelopes of the measured tip resistance profiles. The profiles are very similar and no systematic difference is observed. It is possible to conclude that in the case of unsaturated silt mixtures standard and mini-cone give very similar tip resistances. It is worth noticing that the silt mixtures that were tested in this research are similar to the Calendasco soil in terms of texture.

Second assumption: the tip resistance in pluviated dry sand, according to a number of researches (see as an example [13, 35, 43, 44, 45]), can be expressed by the following equations:

$$Q_c = C_o \cdot \sigma_{v0}^{'C_1} \cdot \sigma_{h0}^{'C_2} \cdot \exp(C_3 \cdot D_R) \qquad (2)$$

$$Q_c = C_o \cdot p_a \cdot \left(\frac{\sigma_m^{'}}{p_a}\right)^{C_1} \cdot \exp(C_2 \cdot D_R) \qquad (2bis)$$

where: Q_c = tip resistance; C_0, C_1, C_2, C_3 — experimental constants; σ'_{v0}, σ'_{h0} — vertical and horizontal effective stress respectively; D_R = relative density as a fraction of 1 and $\sigma_m^{'}$ = mean effective stress. Stresses in eq. 2 are in kPa. Equation 2bis is written in a dimensionless form.

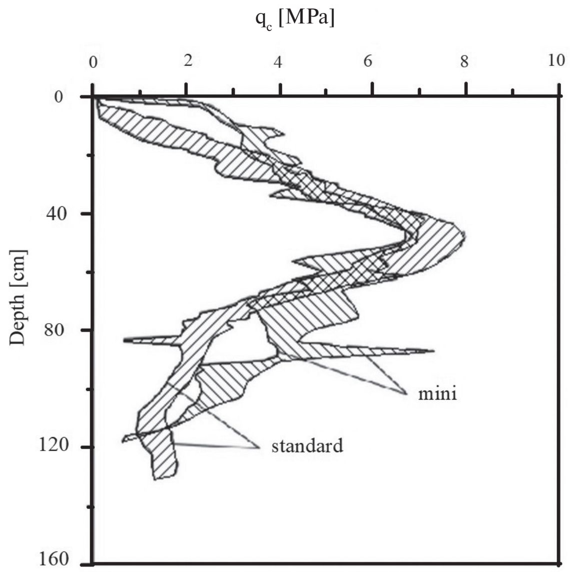

Fig. 1. Upper and lower envelopes of the measured tip resistance profiles for the four standard and the four mini-cone tests carried out at the Calendasco site (Piacenza, Italy)

In practice, it is widely accepted that for dry or saturated clean sands the tip resistance is mainly controlled by relative density, soil type and stress state. As for the stress state, other equations are also available in literature. In an over-simplified approach it is assumed that the tip resistance only depends on the relative density and the vertical effective stress. The results of tests on Ticino sand have been compared to those that can be predicted by means of eq. (2).

In the case of silt mixtures, compacted at a given water content, the boundary stresses are no more representative of the effective stress state which depends on suction (i. e. saturation degree or water content during formation). Moreover, the compaction energy is also a relevant parameter because of the pre-stressing (or pre-straining) of the compacted soil.

It is worth noticing that, according to [71] the relative density is not the relevant index for the compacted state of soil including a large amount of fines content. In this case, Tatsuoka [71] suggests that the degree of compaction, defined for certain compaction energy, is more appropriate. Therefore the influence of the effective stress state in the case of compacted silt mixtures should be defined in a different way.

Third assumption: a ratio between the calibration chamber diameter (D_{CC}) and that of the cone (d_C) equal to 40 is considered acceptable. There is evidence in literature that this type of size effect in sands depends on the boundary conditions and soil dry density (see as an example [33, 35, 44, 45, 52, 70]). Under certain circumstances (very dense sands and zero lateral strain), higher value of the D_{CC}/d_C ratio are necessary in order to consider the CC as an infinite medium. In case of silt mixtures the assumption $D_{CC}/d_C = 40$ seems acceptable. The authors carried out a number of Cone Penetration Tests (CPTs) in a recently constructed river embankment. CPTs were performed at increasing horizontal distances from a Marchetti Flat Dilatometer Test (DMT) blade [53]. The blade was maintained at a given fixed depth and continuously monitored (i. e. the DMT was used as a cell pressure). Fig. 2 shows the locations in plan and section of DMT and Copts' The diameter (d) and depth from ground level (Zv) of the anchor screws is also shown in Fig. 2. Fig. 3 shows the horizontal stress measured by the DMT associated with two of the Copts'. It is clearly seen that when the horizontal distance between the DMT and the CPT is 20 times that of the cone diameter the DMT is no longer sensitive to the passage of the cone.

Fourth assumption: it is considered acceptable that the ratio of the cone diameter to the mean grain size be equal to or greater than 300 [14, 60, 64, 68]. This assumption is necessary to perform tests using a cone having a diameter of only 8 mm in the case of silt mixtures. This hypothesis is not verified for the Ticino sand. It is worth noticing that it is not verified even in the case of standard CPT in Ticino sand. The ratio is about 70 for standard cone and only 16 for the mini-cone.

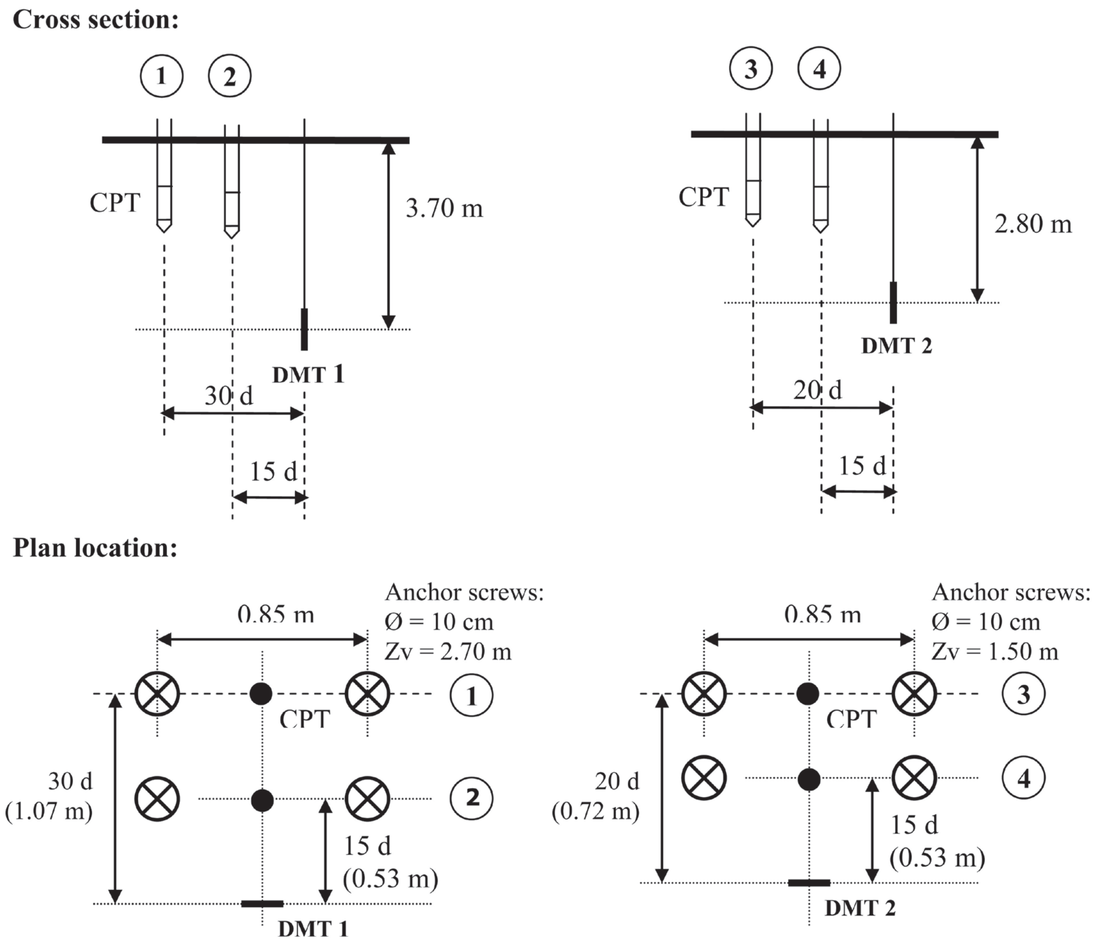

Fig. 2. Cross sections and plan locations of DMT and CPTs

z_{DMT} (m)	z_{CPT} (m)	DMT-A -- 30 d -- (kPa)	z_{CPT} (m)	DMT-A -- 15 d -- (kPa)
	3.00		3.00	
	3.10	86	3.10	85
	3.20	97	3.20	86
	3.30	84	3.30	84
	3.40	84	3.40	87
	3.50	85	3.50	95
	3.60	94	3.60	111
3.70	3.70	82	3.70	122
	3.80		3.80	114
	3.90		3.90	111
	4.00		4.00	97
	4.10		4.10	96
	4.20		4.20	89
	4.30		4.30	86
	4.40		4.40	

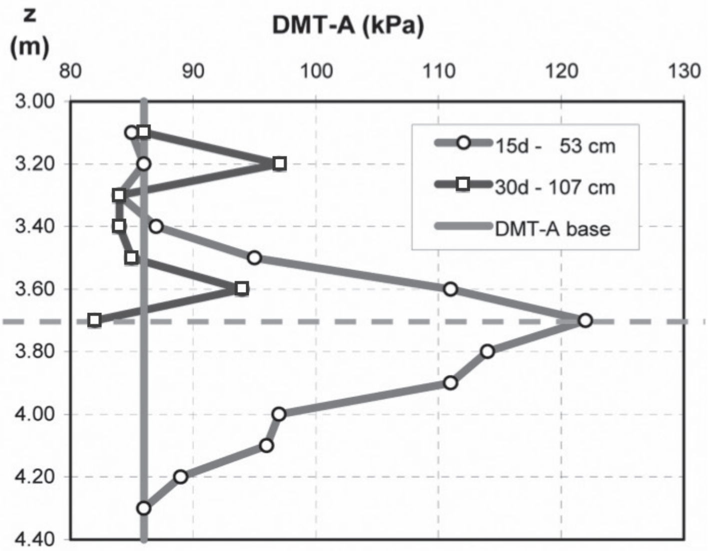

Fig. 3. Horizontal stress measured by the DMT associated with two of the CPTs. Location DMT1 (see Fig. 2). DMT-A: DMT first pressure reading during the penetration of the cone; z: depth

Equipment, materials and testing procedures

The equipment consists of (Fig. 4a):

- a cylindrical aluminum mold with an inner diameter of 320 mm and a height of 210 mm. Lattice membranes are located at the bottom of the mold and all around the internal lateral surface. Air pressure can be inflated inside the membranes in order to apply horizontal and vertical stresses to the sample;
- a stainless steel frame with a lower and upper plate that are connected to each other by means of four stainless steel rods. A locking system is located in the lower plate in order to push up the mold and put it in contact with the upper plate. A nozzle is located in the upper plate for the passage of the mini-cone;
- an electric step motor is used to drive the mini-cone at a constant rate of 20 mm/s. It would be possible to apply different penetration rates but, for the present study, only the standard penetration rate was used. The system uses proximity transducers to automatically stop the penetration when the cone is close to the bottom (30 mm above the base);
- manual air pressure regulators for the vertical and horizontal stresses;
- a mini-cone (8 mm in diameter) with an external sleeve along its full length; The tip resistance was measured by means of a load cell located above the cone. The external sleeve was not in contact with the load cell and therefore the sleeve friction was not measured.

In practice the bottom and lateral surfaces of the CC are flexible boundaries, while the top is rigid.

Ticino sand, and four different silt mixtures (classified as A4 to A6 according to [1]) were used for the testing program. Table 1 summarizes the main characteristics of the silt mixtures (FR, PC, DD, TC).

As for the silt mixtures, the soils were sieved in order to eliminate the fraction with a diameter greater than 2 mm (Fig. 5). The silt mixtures were used for the construction of a new river embankment and for the refurbishment of existing structures.

Ticino sand samples were reconstituted by dry pluviation. In practice the sand was poured into the mold using a funnel that moved over the entire mold surface. This method gave a repeatable relative density of about 40%. The mold was also subject to slight vibrations. This method gave a repeatable relative density of about 60%. Moist tamping would be more appropriate to simulate the behavior of compacted sand fills. In any case, the effects of different sample — reconstitution methods were not investigated in the present study. Moreover, tests on Ticino sand samples were carried out only to validate the equipment, by comparison of the results obtained with the mini-cone in the mini-CC with those available in literature [45].

Table 1

Main characteristics of the tested fine-grained soils: FR, PC, DD, TC.

Soil type	Modified Proctor (ASTM D1557)				Atterberg Limits (ASTM D 4318)			Soil classification		
Abbreviation	γ_{dmax} [kg/m^3]	w_{opt} [%]	e_{opt}	$(Sr)_{opt}$ [%]	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	AASHTO M 145 (1991)	Gs	d_{50} [mm]
FR	2047	9.43	0.33	78	26÷31	18÷24	7÷10	A4÷A6	2.72	0.002÷ 0.025
PC	1950	10.7	0.39	74	25	19	6	A4	2.71	0.085
DD	1820	13.1	0.49	73	31.5	23.5	8	A4	2.71	0.01
TC	1895	12	0.42	77	25	6	19	A6	2.69	0.02

Samples of fine-grained soil were reconstituted in four layers (each 52.5 mm high) using a stainless steel mold with an internal diameter of 310 mm (smaller than that of the CC). The soil was prepared at a given water content and compacted to a given density by applying a vertical pressure to the upper surface of the sample via a loading piston and an upper plate of 300 mm in diameter (i. e. under K_0 conditions). Therefore, each layer was compressed to the desired density by applying a static pressure on the upper surface of the layer. The applied force (pressure) and the associated displacement were measured and recorded. Therefore it was possible to compute the compaction energy per unit volume of soil for each layer and for the whole sample. For each sample the compaction energy was computed according to the following equation:

$$E = \frac{\frac{1}{2}\sum_{i=1}^{4} F_i \cdot \delta_i}{\sum_{i=1}^{4} V_i} \qquad (3)$$

where: F_i — force applied for each layer; δ_i — displacement caused by each applied force; V_i — soil volume of each layer.

After the sample had been reconstituted, it was transferred into the CC. Fig. 4b shows a picture of a sample after extraction from the mold. There was a gap between the sample and the lateral membrane. The CC was then put inside the frame and the locking system was used to push up the CC and put the upper surface of the soil in contact with the upper aluminum plate.

The consolidation stresses were applied in two steps. First the isotropic component of horizontal and vertical boundary stresses was simultaneously applied. After that, the deviatoric component of the consolidation stresses was imposed to the sample.

The penetration test was carried out few minutes after the application of the consolidation stresses.

In practice, the tests (those shown in this paper) were performed under BC1 (Boundary Condition 1, i. e. constant boundary stresses).

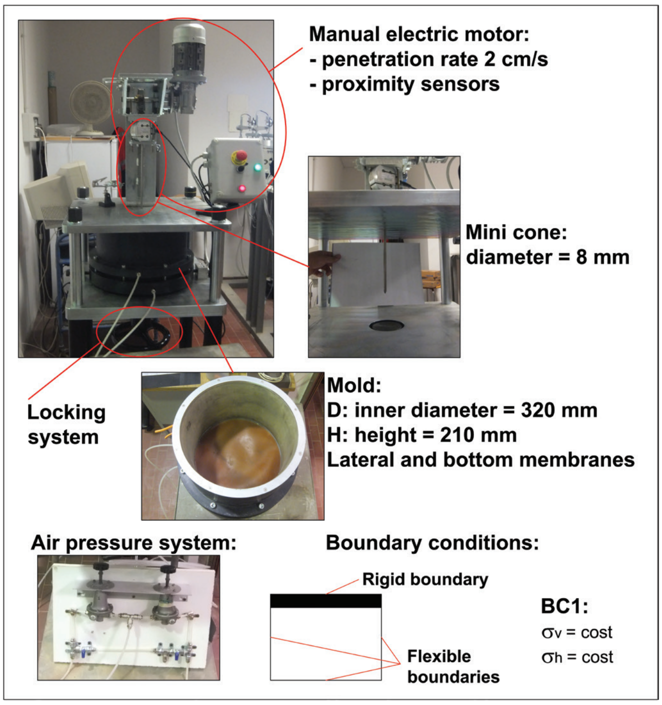

Fig.4a. Equipment

Fig. 4b. Fine-grained soil sample after the CC tests and outline of the occured displacement

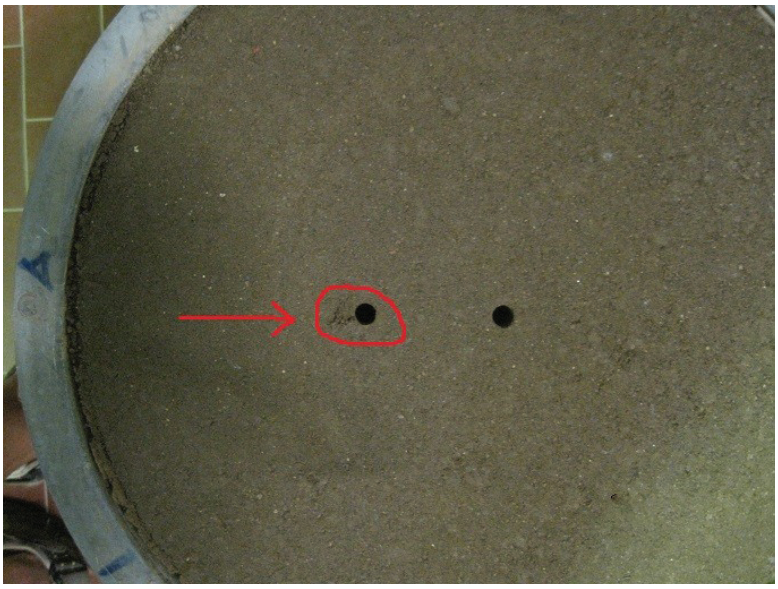

Fig. 4c. Position of two penetration tests repeated on the same sample in the CC:
holes in the upper surface of a TR soil sample after performing two penetration tests in the CC

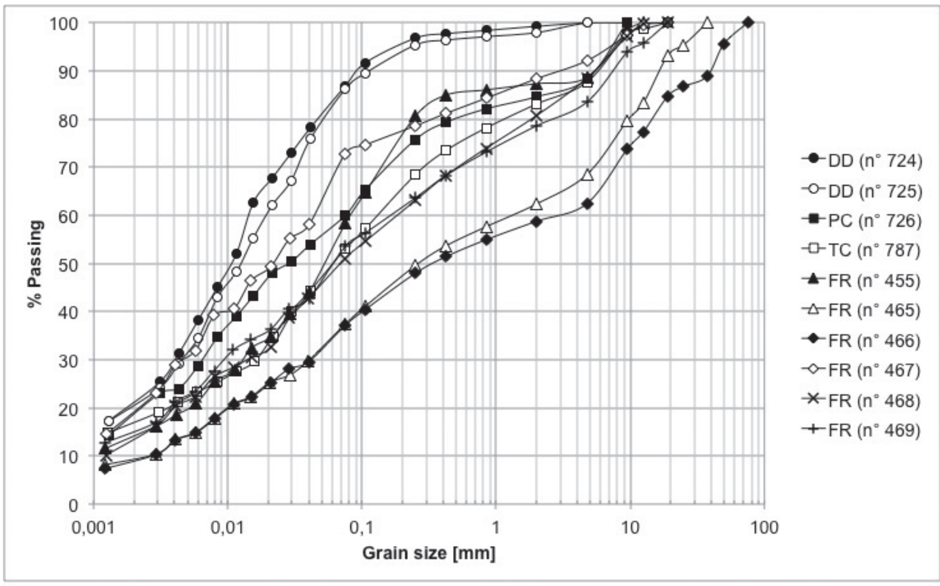

Fig. 5. Grain size distribution curves of the used fine-grained soils. The silt mixtures were sieved in order to eliminate the fraction with a diameter greater than 2 mm

Experimental program and results

Since 2009 many tests were performed in the CC with the mini cone [15, 19, 21, 22, 26, 31, 32, 35, 38, 39, 54, 55, 56, 58, 61, 75] and actually the CC is quite different from its original design and also the experimental procedures were modified. The test results shown in this paper were carried out by means of the above described equipment and following the previously described procedures. Only for the data reported in Table 4, the soil was dynamically compacted within the Proctor Mold (Modified Proctor compaction procedure) and the same Proctor Mold was used as CC (BC3).

Tables 2 and 3 summarize test conditions and results for Ticino sand and fine-grained soils respectively. In particular, Table 2 reports boundary stresses (σ'_v, σ'_h), estimated relative density (D_R), measured average tip resistance and that evaluated by means of equation 2bis.

Table 3 reports soil type; boundary stresses (σ_v, σ_h); sample dry unit weight (γ_d); maximum dry unit weight (Modified Proctor), γ_{dmax}; sample water content (w), optimum water content (Modified Proctor), w_{opt}, compaction energy per unit volume (E); maximum vertical stress applied during sample formation (σ'_{pmax}) and average tip resistance (Q_c).

Table 2

Test conditions and results for Ticino dry sand samples

σ'_v [kPa]	σ'_h [kPa]	Q_c measured [kPa]	Q_c (eq. 2) [kPa]	D_R
50	50	4277	5071	39.7
50	100	6560	6791	39.9
50	150	8269	8272	40,2
50	50	4377	5147	40.2
100	50	4501	6047	40.2
150	50	5772	6851	40.2

Boundary stresses (σ'_v, σ'_h); estimated relative density (D_R); average measured tip resistance (Q_c) and that obtained from eq. 2bis

Table 3

Test conditions and results for fine-grained soils

Soil type	Boundary stresses		Unit weight			Water content		E [MJ/m^3]	σ'_{pmax} [kPa]	Q_c [MPa]
Abbreviation	σ_v [kPa]	σ_h [kPa]	γ_d [kN/m^3]	γ_{dmax} [kN/m^3]	γ_d/γ_{dmax}	W [%]	w_{opt} [%]			
DD	30	30	14.56	17.85	0.82	13.2	13.1	0.395	8224	2.807
DD	50	50	14.56	17.85	0.82	13.2		0.238	6157	1.786
DD	80	80	14.56	17.85	0.82	13.2		0.299	6752	1.512
DD	30	30	16.38	17.85	0.92	13.2		1.324	24474	4.751
DD	50	50	16.38	17.85	0.92	13.2		1.413	24523	4.063
DD	80	80	16.38	17.85	0.92	13.2		1.501	24523	4.990

Soil type	Boundary stresses		Unit weight			Water content		E [MJ/m^3]	σ'_{pmax} [kPa]	Q_c [MPa]
Abbreviation	σ_v [kPa]	σ_h [kPa]	γ_d [kN/m^3]	γ_{dmax} [kN/m^3]	γ_d/γ_{dmax}	W [%]	w_{opt} [%]			
PC	30	30	15.60	19.13	0.82	10.8	10.7	0.62	13731	3.274
PC	50	50	15.60	19.13	0.82	10.8		0.697	14712	3.648
PC	80	80	15.60	19.13	0.82	10.8		0.545	13731	3.850
PC	30	30	17.55	19.13	0.92	10.8		2.407	39627	7.191
PC	50	50	17.55	19.13	0.92	10.8		2.76	40707	7.877
PC	80	80	17.55	19.13	0.92	10.8		2.211	36979	7.603
FR	30	30	18.50	2.05	0.92	12.0	9.43	4.123	46864	6.533
FR	30	30	18.50	2.05	0.92	12.0		3.315	43136	6.535
FR	30	30	18.50	2.05	0.92	12.0		2.938	37465	6.767
FR	30	30	18.00	2.05	0.90	12.0		1.735	22730	3.254
FR	30	30	18.00	2.05	0.90	12.0		1.735	24005	3.568
FR	30	30	18.00	2.05	0.90	12.0		1.828	24400	4.056
FR	30	30	16.00	2.05	0.80	12.0		0.511	8608	1.843
FR	30	30	16.00	2.05	0.80	12.0		0.463	8313	1.736
FR	30	30	16.00	2.05	0.80	12.0		0.475	7823	2.022
FR	30	30	16.00	2.05	0.80	4.0		0.26	10103	2.036
FR	30	30	16.00	2.05	0.80	4.0		0.307	9809	1.479
FR	30	30	16.00	2.05	0.80	4.0		0.346	10790	1.827
FR	30	30	16.00	2.05	0.80	8.0		0.579	15990	3.077
FR	30	30	16.00	2.05	0.80	8.0		0.622	15891	2.533
FR	30	30	16.00	2.05	0.80	8.0		0.564	15303	2.455

Soil type, boundary stresses (σ_v, σ_h), sample dry unit weight (γ_d), maximum dry unit weight (Modified Proctor), sample water content (w), optimum water content (Modified Proctor), compaction energy per unit volume (E), maximum vertical stress applied during sample formation (σ'_{pmax}) and average tip resistance (Q_c).

As for the Ticino sand, a single sample was reconstituted in the laboratory. Indeed, moving the CC in the horizontal plane of about 40 mm in various directions it is possible to perform at least six penetration tests on the same sample. Therefore, a single relative density of about 40 % was considered. On the other hand, different boundary stresses were applied on the same sample. More specifically, firstly the vertical stress was kept constant while the horizontal stress took different values. After that, a second set of stresses was applied by keeping the horizontal stress constant and applying different values of the vertical stress. When the initial boundary stresses of 50 kPa were restored for the second set of tests, the measured average tip resistance was very close to the first measurement. Volume changes, induced by the boundary stresses, were estimated on the basis of literature data [49]. Only the volume changes induced by the isotropic stress component were estimated. The agreement between measured and computed (eq. 2bis) tip resistances seems acceptable, even though a certain scatter is observed (Table 2 and Fig. 6). The low ratio between cone and grains diameters could be a reason for the observed scatter. The following parameters were used to compute the tip resistance by means of eq. 2bis [45]: $C_0 = 23.19$; $C_1 = 0.56$ and $C_2 = 2.97$.

From a multiple — variable linear regression analysis of experimental data, the following values of the parameters of eq. 2 were obtained: $C_0 = 52.4$; $C_1 = 0.22$ and $C_2 = 0.61$. Obviously the C_3 constant could not be assessed as the data referred to a single relative density. Therefore it was assumed $C_3 = 2.97$ [35]. Marginally, it is worthwhile to observe that the exponent C_2 is greater than C_1, i. e. the effect on Q_c of the horizontal stress is greater than that of the vertical one. This result ($C_2 > C_1$) is qualitatively in agreement with the results of a numerical simulations carried out by Arroyo et al. [5] and with experimental evidences [44, 45]. In particular, Arroyo et al. [5] considered a virtual calibration chamber using a three dimensional model based on the discrete-element method and filled with a scaled granular equivalent of the well known Ticino sand. Therefore, the statement that Q_c in sands only depends on the relative density and vertical effective stress is an over simplification.

Samples of fine-grained soils were reconstituted at densities in between 80 and 92 % of the maximum (Modified Proctor) with a water content approximately corresponding to the optimum value. For the FR samples a value of the water content higher than the optimum (9.43 %) was used and a test series at constant density (equal to 80 % of the optimum) and variable water content (4, 8 and 12 %) was also performed.

Therefore, these samples were produced by moist-compaction as in the field compaction.

Fig. 4b shows a sample of fine-grained soil after compaction. The figure qualitatively shows the deformation pattern of the lower surface. It is evident that the

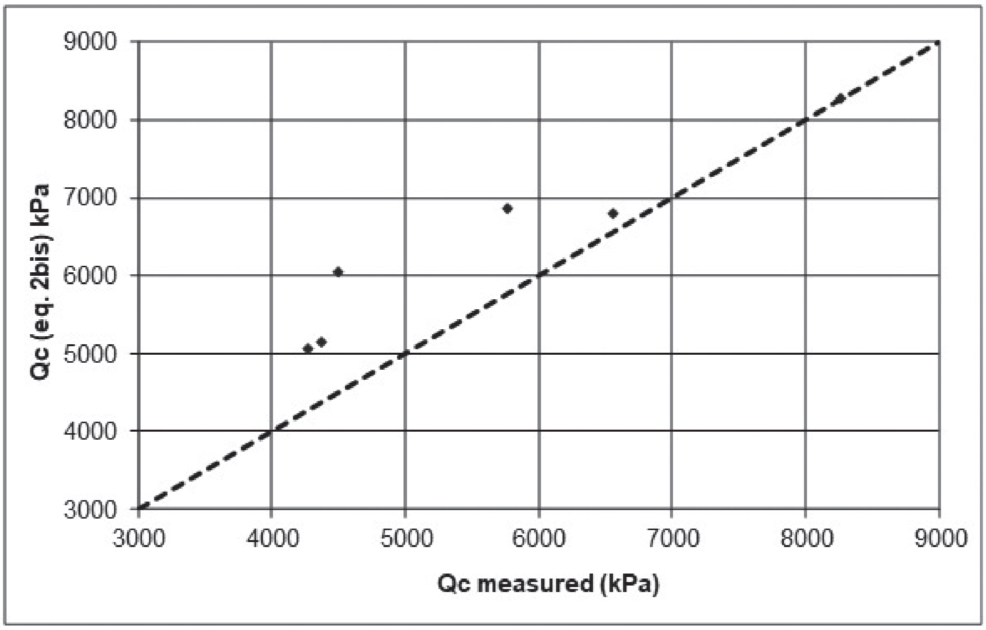

Fig. 6. Average tip resistance from CC tests on dry Ticino sand sample:
measured values vs. those inferred from eq. (2bis)

lower surface, after the application of boundary stresses is no longer plane but exhibit an upward concavity. Measurements of sample heights and diameters (after testing) were performed by means of calipers.

The maximum vertical strain (in the centre of the sample) was of less than 4%. Anyway, the evaluation of current sample volume, after testing, with this method was not considered too much accurate. Therefore, the dry densities reported in the tables refer to the values just after formation.

As for the fine-grained soils it is possible to state that:

1. For a given water content and a given soil a correlation exists between the dry density (γ_d) and the compaction energy per unit volume (E). This aspect can be seen in Fig. 7. FR soil shows a certain scatter especially at higher densities. This scatter could be a consequence of the fact that various batches of FR soil were used and the various batches exhibit small differences.
2. For a given water content and a given soil a correlation exists between the average tip resistance (Q_c) and the compaction energy per unit volume (E). This aspect can be seen in Fig. 8.
3. For a given water content and a given soil a correlation exists between the dry density (γ_d) and the average tip resistance (Q_c). This aspect can be seen in Fig. 9.
4. The effect of boundary stresses seems negligible. Anyway, it could be argued that the applied boundary stresses were never greater than 80 kPa. Therefore in Table 4 are reported few additional data. These data were obtained in a different CC and with a different sample reconstitution method. The samples were dynamically compacted in the Proctor Mold (Modified Proctor procedure) and the same Mold was used as CC (i. e. rigid boundaries and BC3). The results in Table 4 confirm that, even in the case of σ_v ranging in between 25 and 200 kPa the effect of boundary stresses remains negligible. It is supposed that this is a consequence of two facts: effective stresses are mainly controlled by the suction (i. e. water content) and the compaction stresses, applied during sample formation, are several hundreds of times greater than the applied boundary stresses.
5. The last nine rows of Table 3 reports the results of FR soil, compacted at 80% of the optimum and at different water contents (4, 8 and 12%). These data show that a tip resistance of about 1.8–2.0 MPa is obtained for a water content of 12% (greater than the optimum). Also in the case of a water content of 4% (lower than the optimum) a tip resistance of about 1.8–2.0 MPa was measured. Only in the case of a water content of 8% (close to the optimum — 9.43%) a tip resistance of 2.5 to 3.0 MPa was obtained. Therefore, the water content during sample formation has a certain effect on the tip resistance i. e. on the compaction energy which is higher for the case of a water content of 8%.

Table 4
Average tip resistance values as inferred from tests carried out in a CC with rigid top and lateral boundaries and under BC3 condition (i. e. constant vertical stress nd zero lateral strain)

Test number	w [%]	γ_d [kg/m³]	γ_{dmax} [kg/m³]	γ_d/γ_{dmax} [%]	σ_V [kPa]	Q_c [kPa]
1	9.43	1845	2047	90%	25	18200
2					50	18625
3					100	19037
4					150	19751
5					200	21412

In conclusion, it is possible to predict the dry density from the measured tip resistance irrespective of the boundary stresses. The water content during earthwork formation may be also an influent parameter.

The use of compaction equipment measuring the compaction energy represents an alternative to infer the in situ density after an appropriate calibration. It is worthwhile to remember that the compaction energy per unit volume of standard and modified Proctor is respectively equal to 0.59 and 2.69 MJ/m³. Higher compaction energy can be applied in a giratory press [50].

Moreover, the control of the compaction process in the laboratory offers a quantitative evaluation of the soil workability. In fact, Table 3 and Fig. 7 show that some soils are more workable than others.

For example, for FR soil, the maximum compaction pressure or the compaction energy per unit volume that is necessary to obtain a given percentage of the optimum dry density is smaller in comparison with that required in order to compact the PC and DD soils.

In addition, the effect of elapsed time after sample formation and of the variation of the water content was experimentally studied.

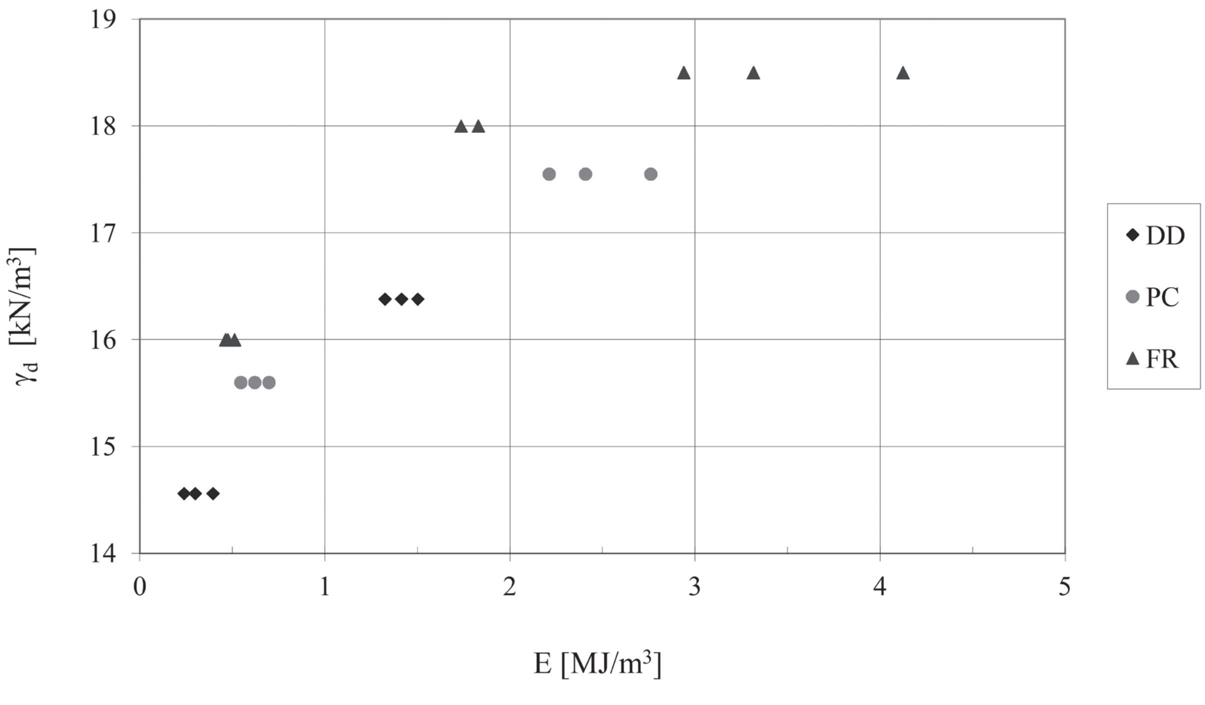

Fig. 7. Partially saturated fine-grained soils: correlation between dry density (γ_d) and compaction energy per unit volume (E)

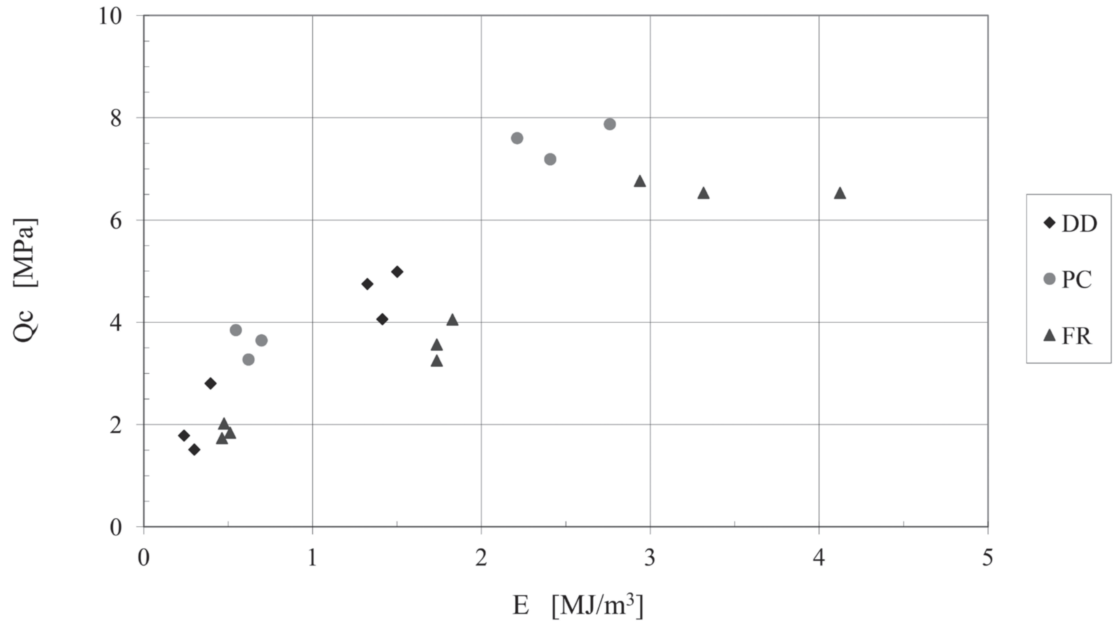

Fig. 8. Partially saturated fine-grained soils: correlation between tip resistance (Q_c) and compaction energy per unit volume (E) for a given water content and a given soil

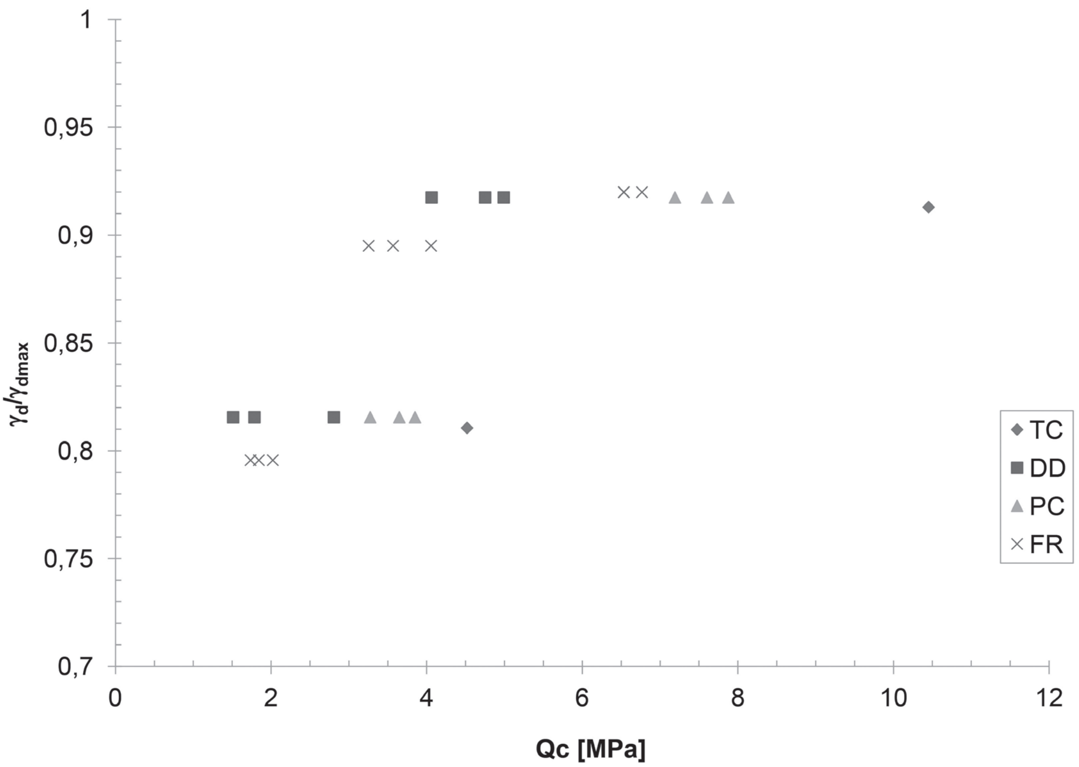

Fig. 9. Partially saturated fine-grained soils: correlation between dry density (γ_d) and tip resistance (Q_c) for a given water content (wopt) and a given soil

Water content and elapsed time effects

The tip resistance variation with water content after the sample formation was studied in the laboratory. A sample of soil was prepared at the optimum water content and a dry density equal to 90 % of the optimum value. Several penetration tests were repeated on the same sample. In fact, it is possible to horizontally move the CC of about 40 mm along all directions and to repeat the penetration tests along different verticals at least 6 times for the same sample. The possibility of performing repeated tests on the same sample was preliminary checked several times. In one occasion tests were repeated during a visit of a research team from MARUM (Center for Marine and Environmental Sciences, University of Bremen, Germany).

The result repeatability (under same test conditions) was really impressive. Fig. 4c shows the holes in the upper surface after performing a series of tests on TR soil.

The tests were carried out at different dates and water contents. The water content decreased with time because of evaporation and was increased by adding water to the sample. Water was sprayed on the top surface in several steps. For each step the water content was increased of about 2.5 %. The penetration test was performed after seven days.

A similar experimental programme was followed using a sample of PC, DD and TC soils.

The tip resistance profiles, measured for each soil during the CC tests, are shown by Figs. 10 (PC), 11 (DD) and 12 (TC). Fig. 13 shows the Q_c (average value between 6 and 15 cm depth) vs. the water content for all the fine-grained soils.

Fig. 14 shows the normalized relation $q_{c\mathrm{LAB}}/(q_{c\mathrm{LAB}})_{\mathrm{opt}}$ vs w/w_{opt} for all the fine-grained soils where $(q_{c\mathrm{LAB}})_{\mathrm{opt}}$ is the tip resistance measured in the CC using a sample compacted at the same density (i. e. 90 % of $\gamma_{d\max}$) at a water content corresponding to the optimum value (w_{opt}).

Tables 5 and 6 summarize, for PC and DD soils, the date of each penetration test, the time elapsed since sample formation, the current water content and the average tip resistance. While for the PC soil the experimental results (Table 5) show that the tip resistance linearly increases with a decrease of the water content and the phenomenon seems perfectly reversible, in the case of DD soil, the data (Table 6) show that the tip resistance also increases with time and not only with a water content decrease. Moreover, in this case the phenomenon is not fully reversible. It is possible to observe a relevant tip resistance increase with the elapsed time nonetheless the water content has been reduced to its initial value.

Table 5

CC tests on a PC soil sample

Test number	Date of the test	Time [Days]	w [%]	Q_c [kPa]
1	22/07/2014	0	10.78	7206
2	07/08/2014	15	10.69	9278
3	05/09/2014	45	10.17	11307
4	19/09/2014	59	9.14	13680
5	02/10/2014	72	11.44	7163

Note: Soil sample: PC; $\gamma_d = 0.9\gamma_{d\max}$

Average tip resistance measured for the same sample, along different verticals, at different dates and water contents.

Table 6

CC tests on a DD soil sample

Test number	Date of the test	Time [Days]	w [%]	Q_c [kPa]
1	16/10/2014	0	12.9	2548
2	27/10/2014	11	15.4	1685
3	03/11/2014	18	17.6	1124
4	10/11/2014	25	17.8	1120
5	21/11/2014	36	13.3	5125
6	05/12/2014	50	10.8	10216
7	22/12/2014	67	7.9	15377

Note: Soil sample: DD; $\gamma_d = 0.9\gamma_{d\max}$. Average tip resistance measured for the same sample, along different verticals, at different dates and water contents.

Therefore, the effect of the elapsed time after sample formation was experimentally studied by performing repeated penetration tests, in the CC, on the same sample over a period of 2 months. The same testing program was repeated using two different material, TR and PE soil samples, in order to compare the results.

The sample water remained constant over the time. Table 8 (TR soil) reports, in the last column, the mass of the CC and of the sample. Measurements of such a mass were taken after each penetration tests. The reported values include 31 kg of CC. The only variations concern the water mass. Initially the mass of the wet soil was 28.025 kg and the initial water mass was 3.025 kg. The water mass variation is of about 0.135 kg, so that the initial water content of 12.1 % reduced to a value of 11.56 %. Similar controls and results are available for PE soil. For PE soil, the water mass variation over a period of time of two months was of 0.205 kg.

Table 7

Main characteristics of the two soils: TR and PE

Abbreviation	Modified Proctor (ASTM D1557)		Atterberg Limits (ASTM D 4318)			Soil classification	Gs
	$\gamma_{d\max}$ [kg/m^3]	w_{opt} [%]	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	AASHTO M 145 (1991)	
TR	1960	12.1	No liquid	No plastic	10.1 %	A3	2.665
PE	1860	10.5	31 %	20.9 %		A4	2.661

The two soil samples were reconstituted at a water content equal to the optimum water content and at a dry density approximately corresponding to the 80 % of the maximum value (Modified Proctor). The main characteristics of the two soils are summarized in Table 7. Test results are shown by Figs. 15 and 16.

Tables 8 and 9 summarize, for each soil, the average tip resistance values measured at different dates. Test results show an almost linear increase of the resistance with the time for both soils (Fig. 17). From the regression analysis of the whole data it is possible to assume an increase of about 40 % of the tip resistance per log cycle of time.

Table 8

TR soil sample: average tip resistance values measured at different dates

Test	Time [Days]	Q_c [kPa]	Mass (kg)
1	7	4253	58.740
2	14	5738	58.730
3	21	5413	58.725
4	28	6461	58.685
5	39	6570	58.650
6	57	6597	58.605

Table 9

PE soil sample: average tip resistance values measured at different dates

Test	Time [Days]	Q_c [kPa]
1	4	4211
2	16	4451
3	28	5492
4	38	5784
5	50	5908
6	60	6044

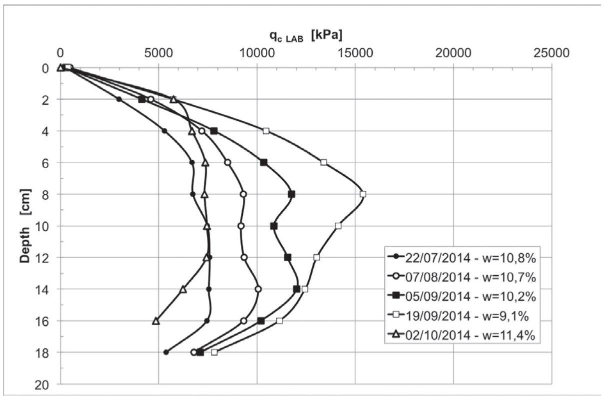

Fig. 10. Tip resistance profiles from calibration chamber tests carried out on the same PC soil sample (sample reconstituted at the dry unit weight equal to the 90 % of the maximum value) at different dates and water contents

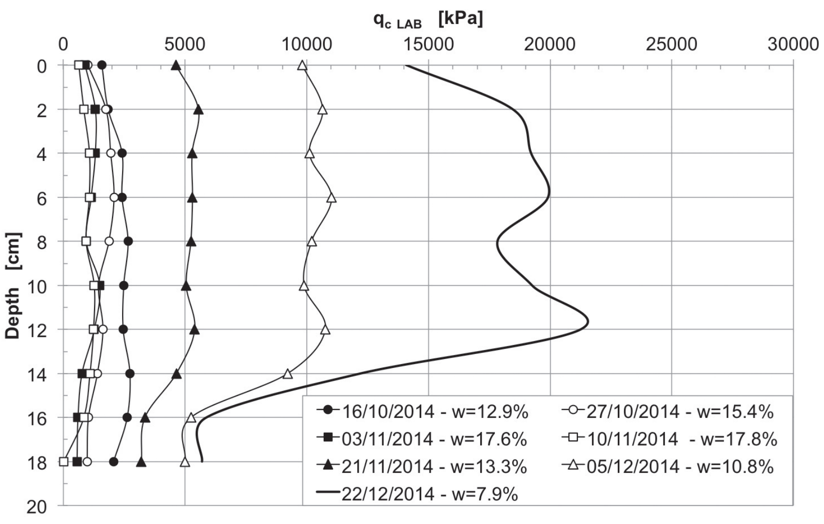

Fig. 11. Tip resistance profiles from calibration chamber tests carried out on the same DD soil sample (sample reconstituted at the dry unit weight equal to the 90 % of the maximum value) at different dates and water contents

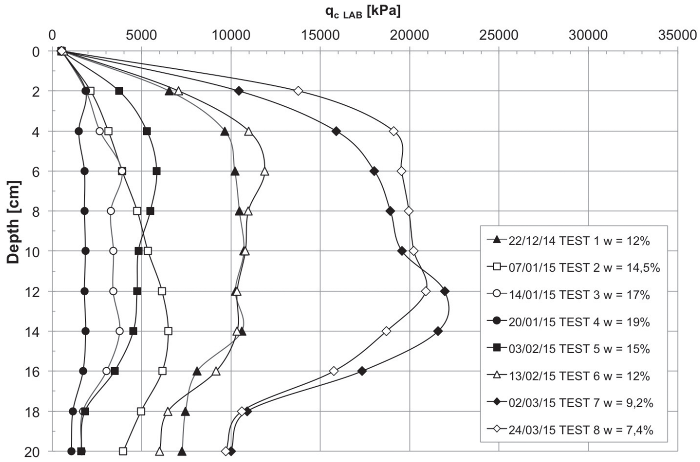

Fig. 12. Tip resistance profiles from calibration chamber tests carried out on the same TC soil sample (sample reconstituted at the dry unit weight equal to the 90 % of the maximum value) at different dates and water contents

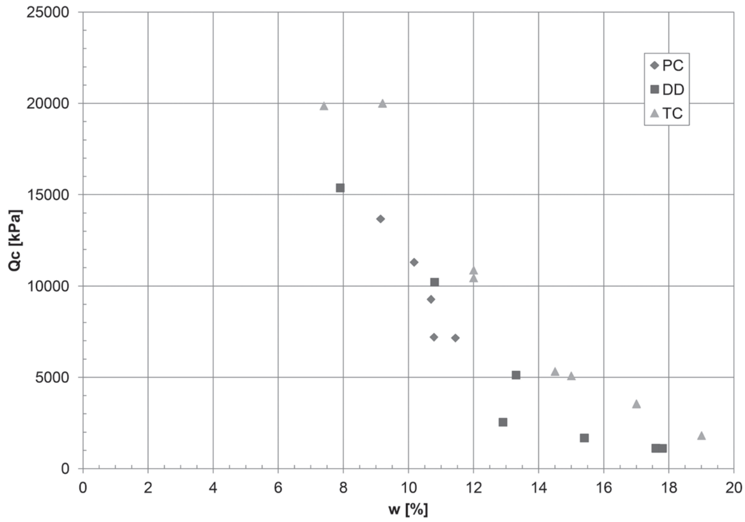

Fig. 13. CC tests at variable water content: average tip resistance vs. water content for all the fine-grained soils

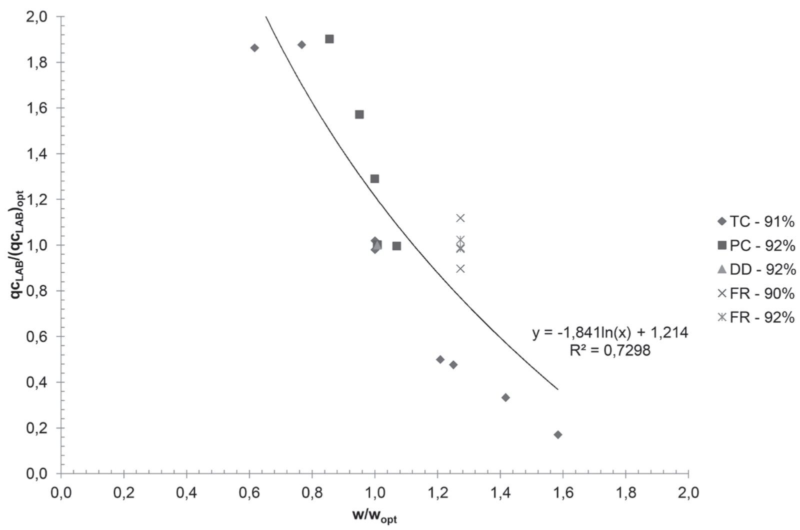

Fig. 14. Relation $q_{c\mathrm{LAB}}/(q_{c\mathrm{LAB}})_{\mathrm{opt}}$ vs w/w_{opt} for all the fine-grained soils: TC, PC, DD and FR and interpolation of the whole data

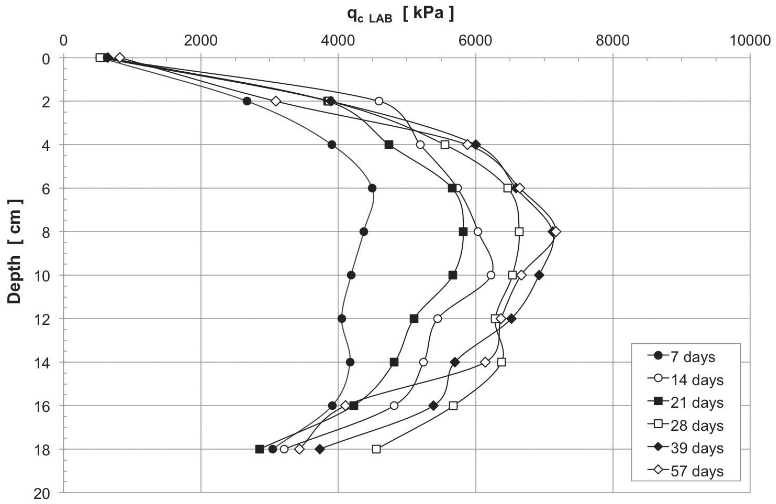

Fig.15. Tip resistance profile from repeated penetration tests, in the CC, on the same TR soil sample over a period of two months

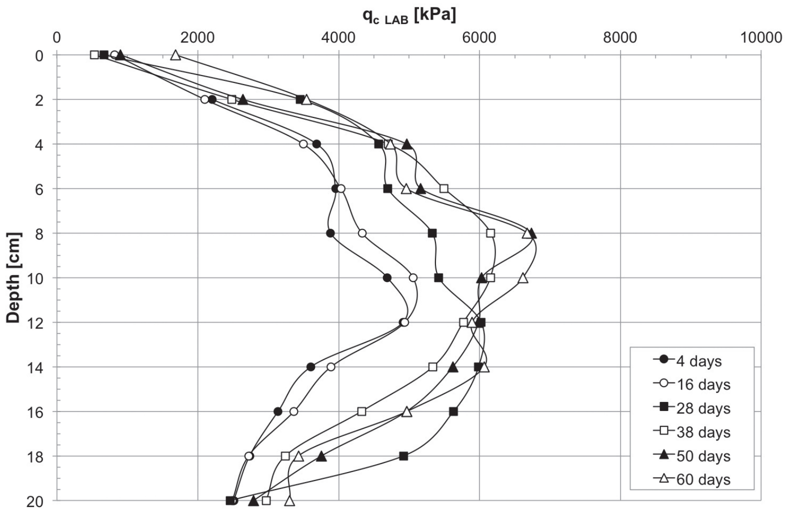

Fig. 16. Tip resistance profile from repeated penetration tests, in the CC, on the same PE soil sample over a period of two months

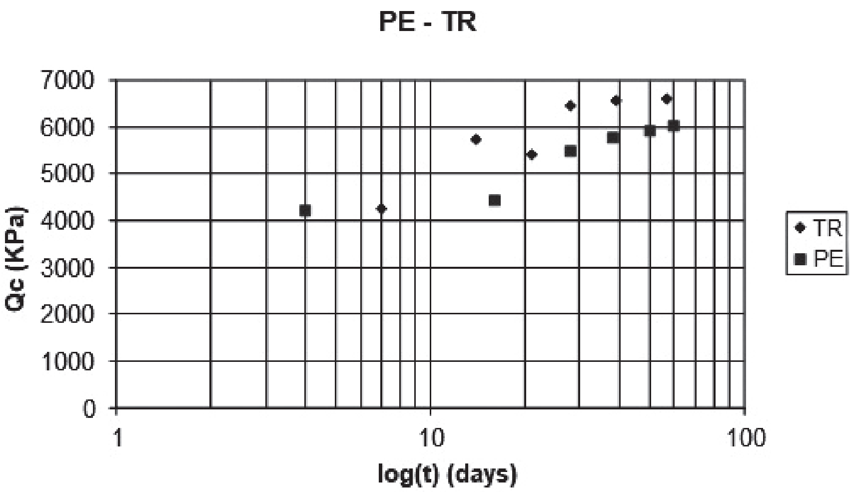

Fig. 17. Average tip resistance versus time for TR and PE soils

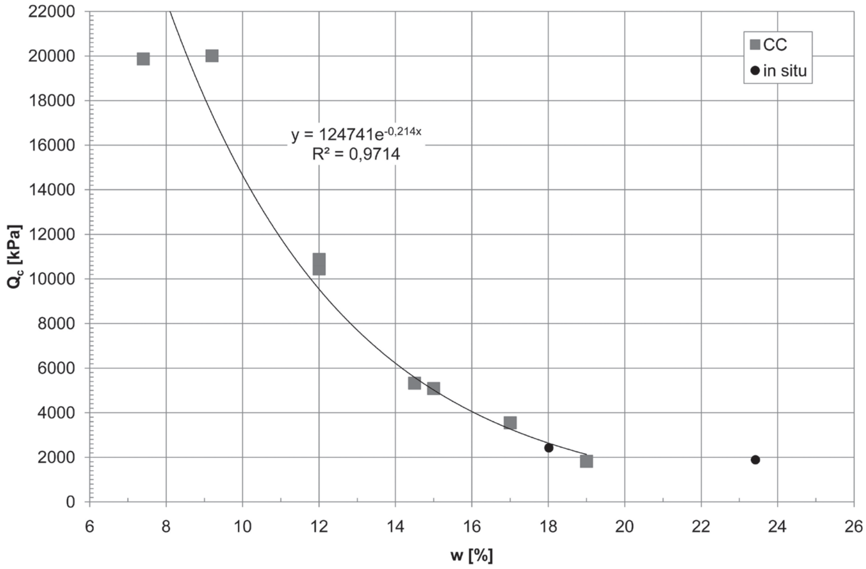

Fig. 18. Average tip resistance values measured in the CC for the tests carried out on the TC soil sample at variable water content and comparison with the average values measured during the in situ control on the levee constructed with the same soil by CPTs

Proposed method and its application

The experimental data previously shown indicate that the tip resistance mainly depends, for a given soil, on the dry density (or compaction degree) and water content after sample formation. More specifically the tip resistance increases of four times when the dry density increases from 80 to 90 % of the optimum. A more important variation of the tip resistance is observed with the water content after sample formation. On the other hand the effect on tip resistance of the water content during sample formation appears less important. The effect of the elapsed time after sample formation can be quantified in an increase of about 40 % per log cycle of time. This effect could be very relevant but it is difficult to evaluate in practice for levees that have been constructed several centuries ago. This aspect requires additional research.

For a practical use of these results it is suggested to determine in the laboratory, for a given soil and a given compaction degree, a normalised curve such as that shown in Fig. 14 or a curve such as shown in Fig. 13. This curve represents the design tip resistance vs. the water content after the earthwork construction. Implicitly, the curve should be determined for the design compaction degree. In other words, this curve is equivalent to the "penetrogramme" of the French standards. For the experimental determination of the design curve it is sufficient to reconstitute a sample of a given soil at a given dry density and water content. On this sample it is possible to repeat the tests with variable water contents after sample formation.

It is worth noticing that, a safety factor of less than 1.0 was obtained from numerical analyses of some cross — sections of the Serchio River levees where a tip resistance of less than 1.0 MPa (about 0.7 Mpa) had been measured [27]. Indeed, for the whole set of tests, performed in CC on various silt mixtures and with a compaction degree ranging in between 80 and 90 % of the optimum, a tip resistance of less than 1.5 MPa was never measured.

Apart the above comment, the method was successfully applied in some real cases (new or refurbished levees).

For example, the method was tested on a newly constructed levee using the TC soil. The levee had a variable height ranging in between 2 to 4 meters. Two CPTs were carried out on the crest of the levee. Two undisturbed samples, specifically cube samples, were taken to preserve as closely as possible the in-place density. They were retrieved very close to in situ CPTs and were subjected to laboratory tests including classification and water content determination. Fig. 18 shows the average tip resistance values, measured in the CC, for the tests carried out on the TC soil sample at variable water content (90 % of the optimum) and compares them with the average values measured during the in situ control by CPTs on the levee constructed with the same soil. From cubic samples a dry density equal to about 90 % of the optimum was obtained. The in situ water content was relatively high because the tests were performed just after biomats wetting. The elapsed time was not taken into account because the tests were performed one month after the earthwork completion. The in situ penetration resistances were consistent with those determined in the CC.

Conclusions

The tests on dry Ticino sand samples as well as those performed at Calendasco suggest that the mini — cone and the mini calibration chamber can represent a reliable physical model of standard CPT in soils. Specifically, the tests on dry Ticino sand confirm that, in the case of granular soils, the tip resistance mainly depends on the relative density and the horizontal effective stress with a minor effect of the vertical effective stress. Therefore, the CPT interpretation, based on the σ'_v is just a necessary over-simplification because of the known difficulty in estimating in situ σ'_h.

Tests on the compacted partially saturated fine-grained soil samples demonstrate that:

- the tip resistance mainly depends on the compaction degree and water content after sample formation. The total boundary stresses are not influent. This could be explained by considering that the effective stress state, in this case, mainly depends on suction and prestressing during compaction;
- the water content during sample formation has a certain influence. However, this effect is not comparable to that of the compaction degree and water content after sample formation. The experimental data of this research suggest that when the water content is close to the optimum value a higher compaction energy is required to obtain a given dry density. The increase on the compaction energy leads in turn to an increase of the tip resistance. This aspect deserve future research;
- for practical purposes, it is suggested to define, for a given soil, a design compaction degree. Therefore it is possible to experimentally determine, for the given compaction degree, the design tip resistance vs. the water content after sample formation. For the experimental determination of this design curve it is sufficient to reconstitute a sample of a given soil at a given dry density and water content. On this sample it is possible to repeat the tests with variable water contents after sample formation;
- the effect of the time elapsed since the sample formation has a great effect;
- (about 40 % per log cycle of time). Also this aspect deserves further research. The most intriguing aspect is how this indication should be applied to earthworks realized centuries ago. For new earthworks, it is suggested to proceed with controls immediately after the work completion.

Acknowledgements

Thanks are due to the District of Lucca for the economical, technical and logistic support. Specifically, the Author would like to thank Mr. G. Costabile and Mr. G. Mazzanti. The Author also would like to thank Mr. Pagani for manufacturing the mini-CC and mini-cone and dr. B. Cosanti for the continuous cooperation.

Notation list

The following symbols are used in this paper:

C_0, C_1, C_2, C_3 — experimental constants
c_v — coefficient of consolidation
D_{CC} — calibration chamber diameter
d — cone diameter
d_C — mini-cone diameter
D_R — relative density
E — compaction energy per unit volume
F_i — force applied to compact each sample layer
K_0 — coefficient of earth pressure at-rest
K_D — horizontal stress index
Q_c — average tip resistance
q_c — tip resistance as inferred from in situ CPT
q_{cLAB} — tip resistance as inferred from calibration chamber tests
$(q_{cLAB})_{opt}$ — tip resistance measured in the CC using a sample compacted at a given density (i. e. 90 % of γ_{dmax}) and at a water content corresponding to the optimum value
V — normalized penetration rate
v — penetration rate
V_i — soil volume of each compacted layer
w — water content
w_{opt} — optimum water content (Modified Proctor)
γ_d — dry unit weight
γ_{dmax} — maximum dry unit weight (Modified Proctor)
ε_i — displacement caused by each applied force during compaction
σ'_h — horizontal effective stress
σ'_v — vertical effective stress
σ_h — horizontal total stress
σ_v — vertical total stress
σ'_{pmax} = maximum vertical stress applied during sample formation

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